Comparison and Selection of Construction Methods for Large-section Tunnels and Analysis of Field Measurements

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Abstract
This paper took Niuzhaishan Tunnel, a double-tube and eight-lane flat tunnel with a large cross section in Pingtan Comprehensive Pilot Zone, as a case study. Numerical analysis was applied to compare the double sidewall heading method, which was adopted in the original design, with the bench excavation method, which was proposed for experiment. The bench excavation method applied in practice was analyzed based on on-site monitoring and measurement data to study the mechanical behaviour of the construction of large-section tunnels. According to the research findings, ground surface settlement happened fast during excavation work and occurred in bursts when the three headings in the upper half-section were excavated; the accumulative settlement volume during this period of time accounted for 85% of the final settlement volume. When the middle heading in the upper half-section was excavated, there were sharp increases in periphery displacement, ground surface settlement and vault settlement; this working condition formed a crucial link in the tunneling process. When the six headings were excavated sequentially, they were mutually affected; in particular, the excavation of the three headings in the lower half-section would exert a large influence on the vault settlement of the three headings in the upper half-section. To ensure safety during construction, the following points should be paid attention to: the drilling footage of each heading should be controlled within 8-10cm; the construction of initial support should keep up with the pace of excavation and its quality should be reinforced; pneumatically placed concrete should be used to reduce rebound rate; the bearing capacity of steel shotcrete should be improved and the construction quality of locking anchor pipes should be ensured to decrease the deformation of steel shotcrete and to improve its stability.

Key words: Flat and Super-Large Span Tunnel, Tunnel, Double-tube and Eight-lane Highway Tunnel, Finite Element Analysis, Monitoring Measurement.

1. INTRODUCTION

With the dramatic increase of traffic flow, the traditional two-way four-lane roads are gradually converted into two-way six-lane or even eight-lane roads. Accordingly, tunnels that traverse through mountains are also turned into six-lane or eight-lane tunnels in order to save land as well as to ensure the orderly connection of highways. Hanjialing Tunnel widened and rebuilt in 2004 became the first double-tube and eight-lane highway tunnel in China. Later, more double-tube and eight-lane highway tunnels are built successively in cities such as Guangzhou, Shenzhen and Fuzhou. The construction methods for such large-section flat tunnels include bench method (Zhao, Liang and Lu, 2014), double sidewall heading method (Xie, Ding and Li, 2010), cross diaphragm (CRD) method (Qi, Tang and Cao, 2014) and center diaphragm (CD) method. China has accumulated some experience in constructing this type of tunnels. For example, Hanjialing Tunnel initially chose the double sidewall heading method and then adopted the bench method in its reconstruction (Chen, Liu and Wan, 2005; Hao, Liu and Wang, 2010; Xi, Zhu and Wang, 2003). Zhou Dingheng Longtoushan Tunnel in Guangzhou and proposed the engineering measure of “controlling large deformation in tunneling” (Zhou, Qu and Cai, 2009; Zhou, Cao and Ma, 2010). Jiang Kun replaced the double sidewall heading method with the CRD method for excavating the Grade-V surrounding rock entrance section of Kuqi No.2 small-clear-distance and large-section tunnel with eight traffic lanes in the second phase of the Fuzhou International Airport Highway project to save construction cost and shorten the duration of the project (Jiang and Xia, 2010; Jiang and Xia, 2012). Since there are only a small number of cases concerning double-tube and eight-lane tunnel engineering and limited
measurement data, scholars in China tend to conduct research through numerical simulation and laboratory experiment. Wu (Wu and Huang, 2006) and Wan (Wan and Hai, 2007) carried out simulation research on excavation methods by building a laboratory scale model; Wang (Wang, Lai and Yang et al, 2009), Li (Li, Lu and Xie, 2013), Su (Su, 2009), Xu (Xu, Xia and Zhu, 2009), Zhang (Zhang, Chen and Cheng, 2013) et al. adopted the finite element (FE) analysis method to simulate and analyze the behavior of different construction methods based on practical engineering projects. It should be noted that double-tube and eight-lane tunnels are of poor stability due to their obvious tabular state and serious stress concentration in the surrounding rock and supporting system. Therefore, theories on single-hole two-lane or three-lane tunnels are not really applicable to the aforesaid double-tube and eight-lane tunnels. At present, research on this type of large-section tunnel are mainly conducted in the form of model test and numerical analysis, and there are only a few studies on construction mechanics based on field measurements. Moreover, there are no explicit guidelines elaborating what kind of construction method should be adopted for a large-section tunnel under a specific surrounding rock condition. Also, the current specifications for the design and construction of road tunnels fail to touch upon double-tube and eight-lane tunnels(Code for design of road tunnel, 2004; Technical code for construction of highway tunnel, 2009). It follows that it is of great engineering value to research and optimize the construction methods by combining numerical analysis with project measurement based on cases of double-tube and eight-lane tunnels with a large cross section.

Targeting the engineering project—the exit section of a double-tube and eight-lane tunnel with a large cross section in Pingtan Comprehensive Pilot Zone, this study employed the FE numerical analysis method to simulate the proposed construction method for the project, and chose the bench excavation method in light of the actual surrounding rock conditions and the results of numerical analyses. On top of that, the dynamic mechanical behavior of the construction of large-section tunnels were analyzed with reference to the monitoring and measurement data obtained on site.

2. EXPERIMENTAL PROCEDURE

Niuzhaishan Tunnel, which is a part of Jinjingwan Avenue in Pintan Comprehensive Pilot Zone between Mile Post K4+335 and Mile Post K5+20336 with a total length of 868 meters, snakes through Niuzhaishan Mountain. The tunnel is designed to be a four-lane tunnel with separate up and down lines. The net span per hole is 18m and the height is 14m. The structural design is shown in Figure 1.

![Figure 1. Typical cross-section of Niuzhaishan tunnel supporting system](image)

The exit section of Niuzhaishan Tunnel is Grade-V surrounding rock. The original design adopted the double sidewall heading method (Scheme A). The tunnel cross section is shown in Figure 2.(a), in which, I, II, III... stand for the sequence of construction: ①excavation of the left sidewall top heading; ②excavation of the left sidewall bottom heading; ③excavation of the right sidewall top heading; ④excavation of the right sidewall bottom heading; ⑤excavation of the middle top heading; ⑥excavation of the middle bottom heading. When excavation started, the surrounding rock condition was found to be good, so Scheme A was tentatively replaced by the bench method (Scheme B).

![Figure 2. Comparison and selection method of excavation](image)
The tunnel cross section is depicted in Figure 2.(b), in which 1, 2, 3... indicate the sequence of construction. During the excavation process, there was a 10m difference between the faces of two adjacent headings in principle. The advantage of Scheme B was that when the three headings in the upper half-section were excavated, the machines could advance through the tunnel, which resulted in higher efficiency and speeded up the construction progress. To compare these two schemes in terms of tunnel displacement and mechanical characteristics, a numerical model was established for simulation analysis.

3. FE ANALYSIS

3.1 Modeling

First of all, a three-dimensional (3D) geological model was developed for the entire site to be analyzed based on the plan and profile of the site for tunnel excavation, as shown in Figure 3. The burial depth of selected cross section was calculated to be 7.5m, and the model dimension was determined according to the scope of influence of tunnel excavation. To be specific, the upper boundary extended to the earth’s surface, the lower boundary was no less than 5 times of the height of the tunnel away from the bottom of the tunnel, and the right and left boundaries were no less than 6 times of the span of the tunnel away from the tunnel entrance. A numerical analysis model was established in GEO5, as indicated in Figure 4. Mohr-Coulomb model was used to simulate the surrounding rock. In the model, the right and left boundaries were subjected to horizontal displacement constraints, the earth’s surface was a free boundary, the bottom was constrained by both horizontal and vertical displacement, and triangular elements were used for meshing.

![Figure 3. 3D stratum model of the tunnel site](image)

![Figure 4. The finite element analysis of a typical section](image)
3.2 Determination of Calculation Parameters

The physical and mechanical parameters of the surrounding rock were selected with reference to exploration data and Code for Design of Road Tunnels (JTG/T D70-2010). The parameters of the anchorage zone of the bolt followed the equivalence principle, which meant they could be simulated by improving the cohesion of the rock mass in the anchorage zone. Table 1. shows the specific parameters.

<table>
<thead>
<tr>
<th>No.</th>
<th>Soil layer</th>
<th>Unit weight $\gamma$ /kN/m$^3$</th>
<th>Elastic modulus $E$ /MPa</th>
<th>Poisson's ratio $\mu$</th>
<th>Cohesion $c$ /kPa</th>
<th>Friction angle $\phi$°</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>stony silty clay</td>
<td>18.5</td>
<td>30</td>
<td>0.36</td>
<td>20</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>residual sandy cohesive soil</td>
<td>18.5</td>
<td>30</td>
<td>0.34</td>
<td>22</td>
<td>17</td>
</tr>
<tr>
<td>3</td>
<td>fully weathered granite</td>
<td>19.0</td>
<td>300</td>
<td>0.32</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>cloddy granite of intensely weathered</td>
<td>20</td>
<td>400</td>
<td>0.30</td>
<td>40</td>
<td>35</td>
</tr>
<tr>
<td>5</td>
<td>granular granite of intensely weathered</td>
<td>20</td>
<td>500</td>
<td>0.30</td>
<td>40</td>
<td>35</td>
</tr>
<tr>
<td>6</td>
<td>moderately weathered granite</td>
<td>22</td>
<td>1000</td>
<td>0.30</td>
<td>70</td>
<td>38</td>
</tr>
<tr>
<td>7</td>
<td>slightly weathered granite</td>
<td>23</td>
<td>3000</td>
<td>0.25</td>
<td>150</td>
<td>45</td>
</tr>
<tr>
<td>8</td>
<td>anchorage zone</td>
<td>23</td>
<td>500</td>
<td>0.28</td>
<td>60</td>
<td>30</td>
</tr>
</tbody>
</table>

3.3 Calculations of Surrounding Rock Stress

Figure 5. and Figure 6. respectively reflect the dynamic changes of the major principal stress of the surrounding rock during the excavation process.

![Contour lines of the maximum principal stresses (scheme A)](image-url)
It can be seen that when the rock and soil mass of each heading was excavated, there was a conspicuous display of stress resilience of the bottom surrounding rock. Meanwhile, a certain degree of stress concentration occurred at the edges of the excavation area, where tensile stress was locally present. Besides, stress concentration was obviously found on the lower right side of the tunnel. When excavation was finished and support was removed, the major principal stress in Scheme A and Scheme B were valued at 280-350kP. For Scheme B, when the second and the third headings were excavated, stress concentration (350-420kPa) arose in the right sidewall bottom heading. Generally, stress concentration in Scheme A was more significant than in Scheme B.

3.4 Structural Deformation and Equivalent Plastic Strain

Figure 7. and Figure 8. respectively show the structural deformation and equivalent plastic strain during excavation in Scheme A and Scheme B.
It can be learned from Figure 7 that in Scheme A, when the right sidewall top heading was excavated, the structural shape after deformation was similar to that of the left sidewall top heading; there was an equivalent plastic strain belt connecting the vertical support of the left sidewall bottom heading with the vertical support of the right sidewall top heading, and the maximum equivalent plastic strain was 0.32%; when the right sidewall bottom heading was excavated, the maximum equivalent plastic strain was 0.38%; when the middle top heading was excavated, the vertical supports at both sides of the heading were subjected to large deformation, and the maximum equivalent plastic strain rose to 0.49%; when the middle bottom heading was excavated, its vertical support underwent large deformation. However, as the soil mass in the middle part of the tunnel that had experienced equivalent plastic strain was removed, there was a decrease in the maximum equivalent plastic
strain, which was measured 0.29%. After all supports were removed, the shape of the tunnel lining basically remained unchanged. The magnitude and distribution of equivalent plastic strain also showed little variation.

It can be judged from Figure 8. that in Scheme B, when the right sidewall top heading was excavated, there was an equivalent plastic strain belt that extended upward at a 45 degree angle on the outside edge of the heading, and the maximum equivalent plastic strain was 0.16%; when the middle top heading was excavated, the vertical support was subjected to large deformation, and the base of the horizontal support was found with large equivalent plastic strain, with a maximum value of 0.48%; when the left sidewall bottom heading was excavated, the maximum equivalent plastic strain was 1.33 %; when the right sidewall bottom heading was excavated, the endpoint at the base of the vertical support of the right sidewall top heading was subjected to large deformation; when all supports were removed, the shape of the tunnel lining basically remained unchanged. On the whole, Scheme A underperformed Scheme B in terms of stress and deformation; as equivalent plastic strain usually occurred at the base of a vertical support, the bearing capacity and strength of the foundation soil of vertical supports should be reinforced.

4. COMPARISON OF CONSTRUCTION ORGANIZATION

To better understand the characteristics of the two schemes, comparisons were made from perspectives such as mechanical characteristics of construction methods, difficulty of construction, speed of construction, technical content and construction cost, as shown in Table 2. Overall, Scheme B outperformed Scheme A. Safety could be ensured on the premises of strict management, good monitoring and measurement, timely feedback of the construction process and effective countermeasures.

<table>
<thead>
<tr>
<th>Scheme</th>
<th>Difficulty</th>
<th>Speed</th>
<th>Technical Content</th>
<th>Comprehensive Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scheme A</td>
<td>After the excavation of top headings, the temporary invert was installed prior to the excavation of bottom headings. As the soil mass underneath the temporary invert was hollowed out, it could no longer bear the load from a large-type excavator or an earthwork transport vehicle that was needed for continuing excavation. In this case, small-sized equipment combined with artificial excavation and transportation was required. The mutual interference between working procedures caused difficulty in construction.</td>
<td>The ongoing excavation of I, II, III headings in the upper half-section asked for small-sized equipment and manual work. Secondary artificial transport of earthwork was needed, which lowered efficiency. Moreover, the transport of steel arch, concrete and other construction materials caused inconvenience.</td>
<td>A lot of manual work was needed, which gave rise to increased labor cost and construction cost. The total cost was 1/3 times higher than Scheme B.</td>
<td></td>
</tr>
<tr>
<td>Scheme B</td>
<td>After the excavation of the three top headings in the upper half-section, the temporary invert was installed prior to the excavation of the bottom headings. After the provision of lower invert, the initial support for the whole tunnel cross section formed a ring closure. The temporary invert in the middle part could be removed at any time, which would not influence the continued excavation of the three top headings in the upper half-section. Due to smaller interference with working procedures, there was lower difficulty in construction.</td>
<td>The ongoing excavation of I, II, III headings in the upper half-section was still carried out by a large-type excavator and an earthwork transport vehicle, resulting in higher efficiency. Moreover, the transport of steel arch, concrete and other construction materials was convenient.</td>
<td>The construction was mainly performed by large machines with less input of manual work, which was more economical.</td>
<td></td>
</tr>
</tbody>
</table>
5. COMPARISON OF MEASUREMENTS AND MODELING RESULTS

Scheme B was applied in actual construction. The large cross section of the tunnel under study could cause higher construction risk. To ensure the stability of the surrounding rock during construction and to provide guidance for the recurrent measurement of design and construction by judging the stability of the surrounding rock, the monitoring content for this study included ground surface settlement, vault settlement and periphery displacement. The layout of each item to be monitored abided by the principle of “selecting typical sections for tests”, as shown in Figure 9.

![Figure 9. Representative cross sectional view of instruments arrangement](image)

5.1 Ground Surface Displacement

The ground surface settlement observation point GS-4 was chosen for comparison between FE simulation results and field measurements. Figure 10. shows the development trend of GS-4 during the excavation process.

![Figure 10. ground subsidence with the progress of the tunnel excavation](image)
It can be seen from the field data that the monitoring results shared the similar settlement varying pattern with the simulation results. As excavation proceeded, deformation fell into 6 stages: ① On November 12, 2012, the excavation of the left sidewall top heading and the right sidewall top heading was carried out, and support was installed for the advance anchor bolt. This stage witnessed a relatively slow rate of ground surface settlement and a slight change of accumulative settlement volume; ② From the 44th day on, the excavation faces of the left sidewall top heading and the right sidewall top heading approached to the monitoring point, accompanied by an accelerated settlement rate and rapidly increased accumulative settlement volume; ③ On the 59th day, the excavation of the middle top heading began. Accordingly, settlement speeded up and its rate was positively correlated with the drilling footage of the middle top heading; ④ On the 102nd day, the excavation of the three headings in the upper half-section was completed, by which time, the settlement rate had reached its maximum value; ⑤ Soon afterwards, the excavation of the left sidewall bottom heading and the right sidewall bottom heading was carried out, and the excavation of the middle bottom heading started on the 108th day. The settlement volume at this stage basically remained unchanged; ⑥ On the 142nd day, the excavation of the whole tunnel was completed, and the initial support was finished. Ground surface settlement plateaued, and the final settlement volume was measured around 58mm at its maximum.

All in all, ground surface settlement appeared more frequently during the excavation of the middle heading in the upper half-section, with the accumulative settlement volume accounting for 85% of the final settlement volume. It was suggested that the construction quality of the vertical support be reinforced to effectively control deformation.

5.2 Vault Settlement

Figure 11. shows the changes of vault settlement as a result of the excavation process.

![Figure 11. The vault crown settlement with the progress of the tunnel excavation](image)

The actual construction process was as follows: the excavation of the left sidewall bottom heading started on the 28th day, the excavation of the right sidewall bottom heading started on the 43rd day, and the excavation of the middle bottom heading started on the 52nd day. Figure 11. indicates that the measured vault settlement basically agreed with the numerical analysis results in tendency. The settlement of vaults on both left and right sides was characterized by staged distribution. Besides, the approximate points of contraflexure on the time curve were all close to the time points when each heading was excavated, which suggested that the headings were mutually affected. It should be noted that there was a point signaling abrupt changes in settlement when excavation of headings in the lower half-section began. At this moment, settlement accelerated, and the accumulative settlement volume after this point made up over 50% of the total settlement volume. This indicated that the excavation of the headings in the lower half-section exerted a large influence on the excavation of the headings in the upper half-section. Thus, it is advisable that during the excavation of the headings in the lower half-section, the initial support should be timely followed up. Meanwhile, real-time monitoring of vault settlement should be performed to detect problems and take measures in time.

5.3 Periphery Displacement

Figure 12. shows the varying pattern of periphery displacement during the excavation process. It can be seen that the monitoring results and the FE simulation results differed remarkably with each other in the earlier stage of construction, but gradually became consistent in the latter stage. On the 18th day, there was an upsurge in periphery displacement, because during this period of time, the three headings in the upper half-section were constructed simultaneously, accompanied by a vehement release of stress from the surrounding rock. It was thus clear that the simultaneous construction of the three headings in the upper half-section created a hazardous condition. To control deformations and reduce potential safety hazards, it is strongly suggested that construction be carried out separately, and that the faces of adjacent headings keep an appropriate distance in between.
To sum up, Scheme B was generally a safe construction method to be applied. However, the following measures should be adopted to control deformation: (1) the drilling footage of each heading should be controlled within 8-10m; the excavation of each heading was divided into the upper bench and lower bench, whose length should be 3-5m; (2) the initial support should keep up with the pace of excavation; (3) the construction quality of the initial support should be reinforced; pneumatically placed concrete should be used to reduce rebound rate; the bearing capacity of steel shotcrete should be increased to decrease its deformation and improve its stability; (4) the construction quality of the locking anchor pipes should be paid attention to; (5) real-time monitoring should be enhanced to detect problems and take measures in time.

6. CONCLUSIONS

This paper took Niuzhaishan Tunnel, a double-tube and eight-lane flat tunnel with large cross section in Pingtan Comprehensive Pilot Zone, as an example to compare the dynamic measurements during construction with the calculations derived from FE numerical analysis, and came to the following conclusions:

(1) Through FE modeling and calculation, it was found that there was an obvious stress concentration effect on the left and right sides of the final tunnel excavated in the optimized Scheme B, but numerical value of the stress was small. From the perspective of numerical analysis, the surrounding rock was generally safe and stable.

(2) To enlarge the working space for mechanized construction, simplify the management of working procedures and expedite the construction progress, this engineering project optimized the double sidewall heading method and made several modifications to the working procedures, namely, converting the conventional practice of “excavating two headings on the left side first, and then two on the right side, and finally two in the middle part” into “excavating three headings in the upper half-section and then three in the lower half-section”. Practice proved that this method was feasible.

(3) According to the comparison between the monitoring and measurement results and the simulation results, ground surface settlement, vault settlement and periphery settlement followed the same variation trend. Ground surface settlement mainly occurred during excavation and appeared more frequently when the three headings in the upper half-section were excavated; the accumulative settlement volume during this period of time accounted for over 85% of the final settlement volume. When the middle heading in the upper half-section was excavated, there were sharp increases in periphery displacement, ground surface settlement and vault settlement; this stage formed a crucial link in the tunneling process. The excavation of the three headings in the lower half-section would exert a significant influence on the vault settlement of the three headings in the upper half-section.

(4) To ensure safety during construction, the following points needed to be carefully attended when Scheme B was adopted for excavation: the drilling footage of each heading should be controlled within 8-10cm; the construction of initial support should keep up with the pace of excavation and its quality should be reinforced; pneumatically placed concrete should be used to reduce rebound rate; the bearing capacity of steel shotcrete should be improved and the construction quality of locking anchor pipes should be ensured to decrease the deformation of steel shotcrete and to improve its stability.

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Revista de la Facultad de Ingeniería U.C.V., Vol. 32, N°2, pp. 84-94, 2017


